APPENDIX B

RAPID INFILTRATION DESIGN EXAMPLE

B.1 Introduction

The design example described in this appendix is intended to demonstrate only the RI design procedures described in Chapter 5; therefore, components that are common to most wastewater treatment systems, such as transmission systems and pumping stations, are described but not designed in detail. However, a cost estimate and an energy budget are developed for the entire system.

B.2 Design Considerations

B.2.1 Design Community

Community B is located in the southeastern United States on the Coastal Plain. The area in which the community is located is characterized by relatively flat areas lying between numerous creeks and swamps that drain into North Creek. One of these creeks, South Creek, borders the northeast edge of the community. The elevation of Community B is 45.7 m (150 ft); near the community, elevations range from 42.7 to 54.9 m (140 to 180 ft).

B.2.2 Wastewater Quality and Quantity

The design average daily flow is 6,060 m^3/d (1.6 Mgal/d) and the design peak flow is 9,090 m^3/d (2.4 Mgal/d).

Expected wastewater characteristics under design flow conditions are presented in Table B-1. Wastewater is essentially domestic in character and expected concentrations of trace elements and organics are low.

TABLE B-l
PROJECTED WASTEWATER CHARACTERISTICS

Parameter	Value
BOD_5, mg/L	175
Total suspended solids, mg/L	150
Total nitrogen, mg/L	50
Ammonia nitrogen (as N), mg/L	20
Total phosphorus (as P), mg/L	10
pH, units	6.9

B.2.3 Existing Wastewater Treatment Facilities

The existing treatment facilities provide primary treatment, and treated wastewater fails to meet present discharge requirements. The facilities are old and would require significant repairs and additions to produce treated water that would meet all discharge requirements.

B.2.4 Discharge Requirements

Discharge requirements for surface waters are presented in Table B-2. The ammonia nitrogen limit during summer months is intended to prevent ammonia toxicity to fish. The inhibited test for carbonaceous BOD does not measure nitrogenous BOD. The test is often specified for systems that nitrify wastewater, because such systems tend to have higher BOD_5 concentrations although the water quality is equivalent.

TABLE B-2
SURFACE WATER DISCHARGE REQUIREMENTS

Parameter	North Creek	South Creek
BOD_5, mg/L (inhibited test for carbonaceous BOD)	30	20
Dissolved oxygen, mg/L	5	5
pH	6-9	6-9
Total suspended solids, mg/L	30	20
Fecal coliforms, MPN/100 mL	200	200
Ammonia nitrogen (as N), mg/L (May-October only)	2	2

B.2.5 Climate

Average temperature and precipitation in Community B were obtained from local climatological data and are shown by month in Table B-3. A rainfall frequency distribution curve, developed from 26 years of recorded data, indicates that the wettest year in 10 yields 137 cm (54 in.) of precipitation in Community B. The average total annual precipitation (rain plus snow) is 111 cm (43.7 in.).

TABLE B-3
AVERAGE METEOROLOGICAL CONDITIONS

Month	Temperature, °C	Precipitation, cm	
		Rain	Snow[a]
Jan	8.6	6.71	0.25
Feb	9.3	8.05	0.51
Mar	12.6	9.24	1.02
Apr	17.5	9.17	0.00
May	22.2	7.34	0.00
Jun	26.0	10.87	0.00
Jul	27.0	15.85	0.00
Aug	26.6	11.61	0.00
Sep	23.8	10.41	0.00
Oct	18.3	5.54	0.00
Nov	12.6	5.87	Trace
Dec	8.4	7.77	0.76
Year	17.8	108.43	2.54

a. Water equivalent.

B.3 Site and Process Selection

Community B contacted landowners within a 4 km (2.5 mile) radius of the existing treatment facilities to determine their interest in leasing or selling their property for land treatment. Five potential sites were identified during Phase 1 of the planning process and screened in accordance with the procedure in Chapter 2. Two of the sites were available for purchase and had soils suitable for RI (Sites 1 and 2 on Figure B-l). One of these two sites (Site 2) and the three remaining sites had enough land to be suitable for SR. None of the soils in the area were suitable for OF (Table B-4). Therefore, OF was eliminated from consideration as a viable alternative.

During phase 2 of the planning process, field investigations were conducted at each of the five sites. Based on the field investigations, preliminary design criteria and cost estimates were developed. This analysis indicated that the two RI alternatives were more cost effective than any of the SR alternatives and lower in total present worth than the best conventional secondary treatment and discharge alternative. The preliminary analysis also indicated that an RI facility at Site 1 would be slightly less expensive than an RI system at Site 2. For these reasons, the alternative selected by Community B was RI at Site 1.

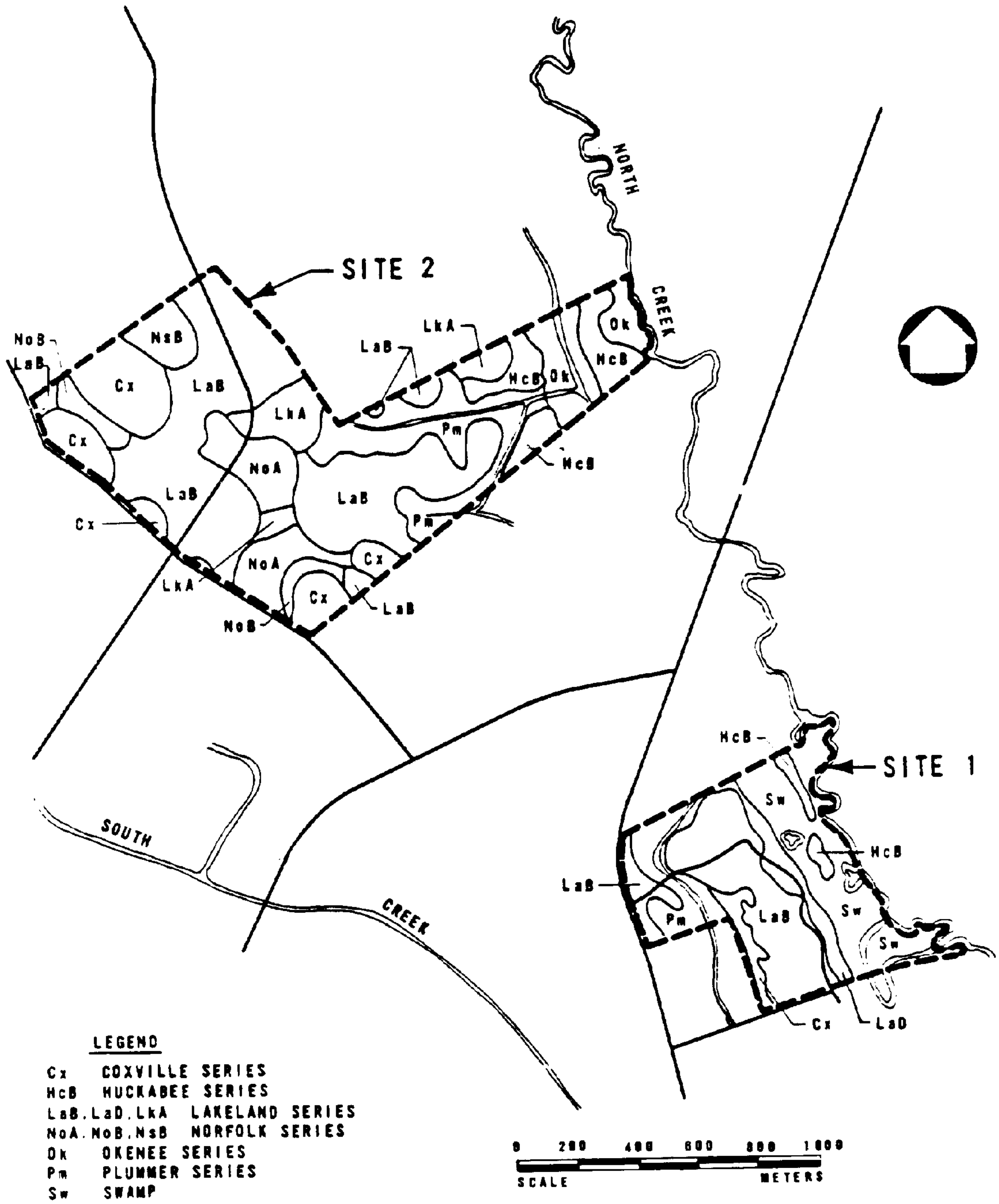

FIGURE B-1
SOILS MAP, SITES 1 AND 2

TABLE B-4
GENERAL SOIL CHARACTERISTICS, SITES 1 AND 2

SCS soil series	Depth, cm	USDA texture	Estimated perme-ability, cm/h	Depth to seasonal high water table, m	Drain-age class	Available water capacity, cm/m	Shrink-swell potential	Structure	pH	Limitations for rapid infiltration
Coxville (Cx)	0-30	Fine sandy loam to sandy loam	0.13-0.51	0	Poor	13	Low	Crumb	5.1-5.5	Fine texture, low per-meability; high water table; poor drainage; moderate shrink-swell
	30-91	Sandy clay loam to sandy clay	0.13-0.51	--	--	12	Low-moderate	Sub-angular blocky	5.1-5.5	--
Huckabee (HcB)	0-41	Sand to loamy sand	25	1.5+	Exces-sive	5.8	Low	Crumb	5.6-6.0	--
	41-91	Loamy sand to sand	5.1-13	--	--	5.8	Low	Crumb	5.1-5.5	--
Lakeland (LaB, LaD, LkA)	0-20	Sand	25	1.5+	Exces-sive	5.8	Low	Crumb	5.1-5.5	--
	20-137	Sand to loamy sand	6.4-13	--	--	5.8	Low	Struc-tureless	5.6-6.0	--
Norfolk (NoA, NoB)	0-76	Loamy sand	6.4-13	0.9	Well	6.7	Low	Crumb	5.6-6.0	High water table
	76-107	Sandy loam	6.4-13	--	--	6.7	Low	Sub-angular blocky	5.6-6.0	--
Norfolk (NsB)	0-33	Sandy loam	2.0-6.4	0.9	Well	8.3	Low	Crumb	5.6-6.0	Fine texture; low per-meability; high water table
	33-112	Sandy clay loam	0.13-0.51	--	--	8.3	Low	Sub-angular blocky	--	--
Okenee (Ok)	0-33	Loam	2.0-6.4	0	Poor	12	Moderate	Crumb	5.1-5.5	Fine texture; low per-meability; high water table; poor drainage; moderate shrink-swell
	33-107	Sand loam to sandy clay loam	0.51-2.0	--	--	14	Low	Sub-angular blocky		--
Plummer (Pm)	0-28	Loamy sand	2.0-6.4	0	Poor	6.7	Low	Crumb	5.1-5.5	Low permeability; high water table; poor drainage
	28-81	Loamy sand	0.51-2.0	--	--	5.7	Low	Sub-angular blocky	5.1-5.5	--
Swamp (Sw)	0-91	Variable	Variable	0	Poor	Variable	Low	Variable	5.1-5.5	High water table; poor drainage

B.4 Site Investigations

The selected site for RI is 2.4 km (1.5 miles) from the existing wastewater treatment facilities. The site contains 48 ha (120 acres) of land and was covered with brush and trees. Near North Creek, the ground surface drops vertically about 6 m (20 ft), forming a relatively steep bluff as indicated in Figure B-2. West of the bluff, elevation varies less than 0.6 m (2 ft).

B.4.1 Soil Characteristics

As indicated by Figure B-1 and Table B-4, the soils at Site 1 that are best suited for RI are the Lakeland sands (LaB and LaD in Figure B-1). These permeable soils are found at Site 1 only near the center of the site. Thus, RI is potentially feasible only in a limited portion of Site 1. Because it would have cost Community B as much to buy only the land needed for the treatment system as to buy the entire site (the unused portion of the site being mostly swamp and therefore undevelopable), acquisition of the entire site was necessary.

To verify that Site 1 has adequate soil depth and depth to ground water for RI, and to ascertain the absence of shallow, impermeable soil layers, nine test holes were drilled as shown in Figure B-2. A typical boring log from the investigation is presented in Table B-5. At this particular test hole, the presence of ground water at a depth of 3.2 to 3.5 m (10 to 11 ft) and an impermeable clay layer at 6.5 m (21 ft) means that percolation could occur only to a depth of about 3.2 to 3.5 m (10 to 11 ft) and that the flow of water below this depth is primarily horizontal rather than vertical.

TABLE B-5
TYPICAL LOG OF TEST HOLE

Depth, m	USDA texture	Remarks
0-1	Loamy sand	--
1-2	Sandy loam	--
2-2.2	Loamy sand	With thin silt lenses
2.2-3.2	Sand	--
3.2-3.5	Sand	Ground water table
3.5-6.5	Sand	Saturated
>6.5	Clay	Impermeable

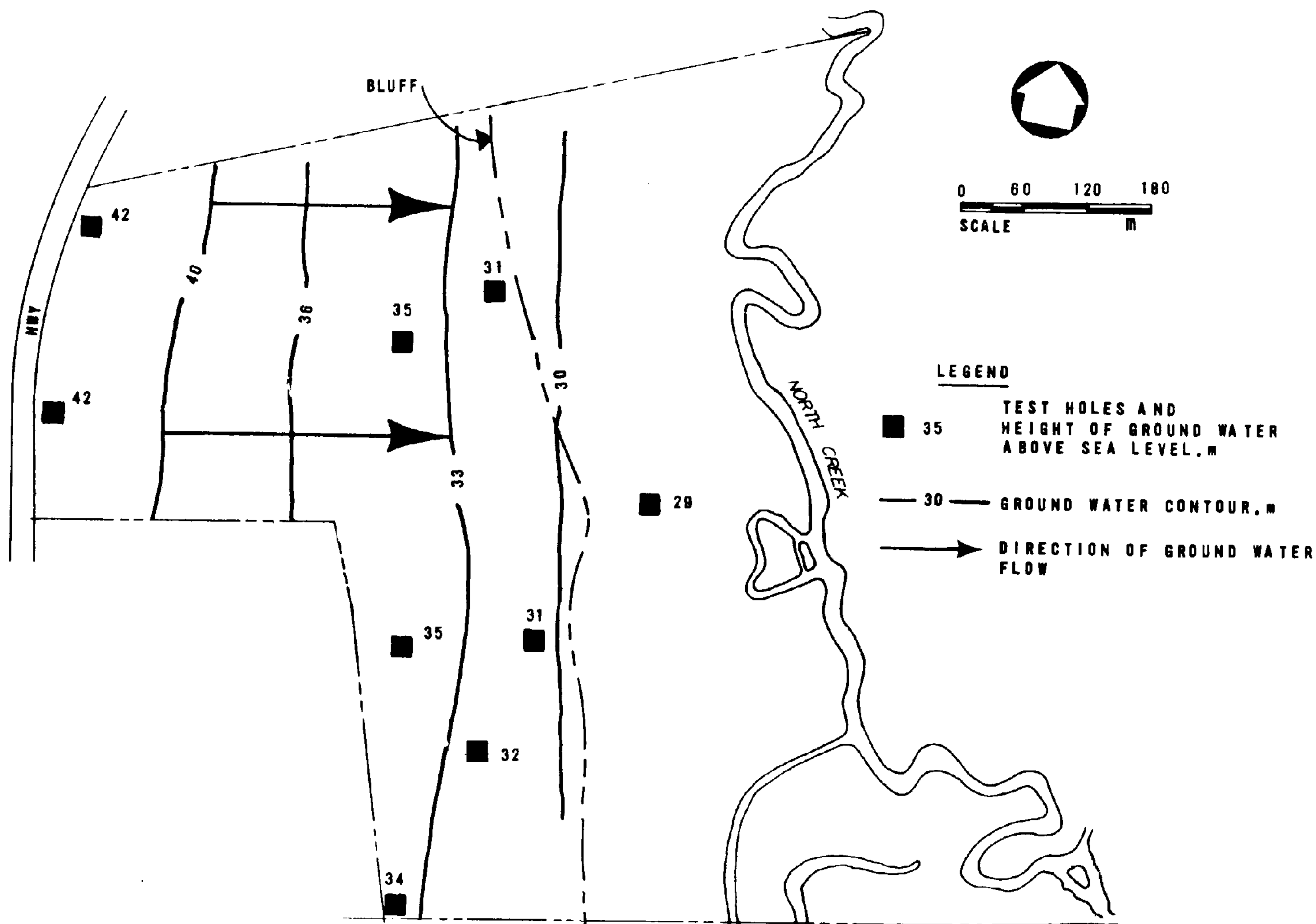

FIGURE B-2
GROUND WATER CONTOURS

B.4.2 Ground Water Characteristics

At the selected site, the depth to ground water ranges from 1.5 to 4.6 m (5 to 15 ft) and is typically 3 m (10 ft). The ground water aquifer is 1.5 to 4.6 m (5 to 15 ft) thick and is underlain by impermeable clay. The clay layer prevents deep vertical percolation and causes the ground water to flow laterally toward North Creek, as indicated by the approximated ground water contours shown in Figure B-2. Because of the shallow ground water table, there is a potential for mounding of the percolate and underdrains must be considered. Horizontal hydraulic conductivity in the aquifer was measured using the auger hole technique (Section 3.6.2.1) and averaged 3.4 m/d (11 ft/d).

Furthermore, although ground water quality is adequate for water supply purposes, the aquifer is too thin to allow production wells to extract ground water economically. The closest domestic water supply well to the RI site is 1.6 km (1 mile) southwest and upgradient of the site. This well and others in the area pump water from depths of 90 to over 150 m (300 to over 500 ft). Thus, the shallow aquifer underlying the area to be used for RI and between the RI area and North Creek will not be used as a potable water source. Current ground water quality data are presented in Table B-6.

TABLE B-6
GROUND WATER QUALITY

Parameter	Concentration
pH, units	6.8
Specific conductance, μmhos	120
Nitrate nitrogen, mg/L	8.4
Fecal coliforms, MPN/100 mL	0

B.4.3 Hydraulic Capacity

Basin infiltration tests at the selected site were performed with clear water using 3.6 by 3.6 by 0.5 m (12 by 12 by 1.5 ft) basins filled to a depth of 22 to 30 cm (9 to 12 in.). Because the soil and ground water characteristics were generally uniform throughout the site, only two basin infiltration tests were performed. If the results of these two tests had conflicted, additional tests would have been conducted. Results from one of the two infiltration tests are plotted in Figure B-3. As shown in this figure, the resulting limiting infiltration rate at this basin was

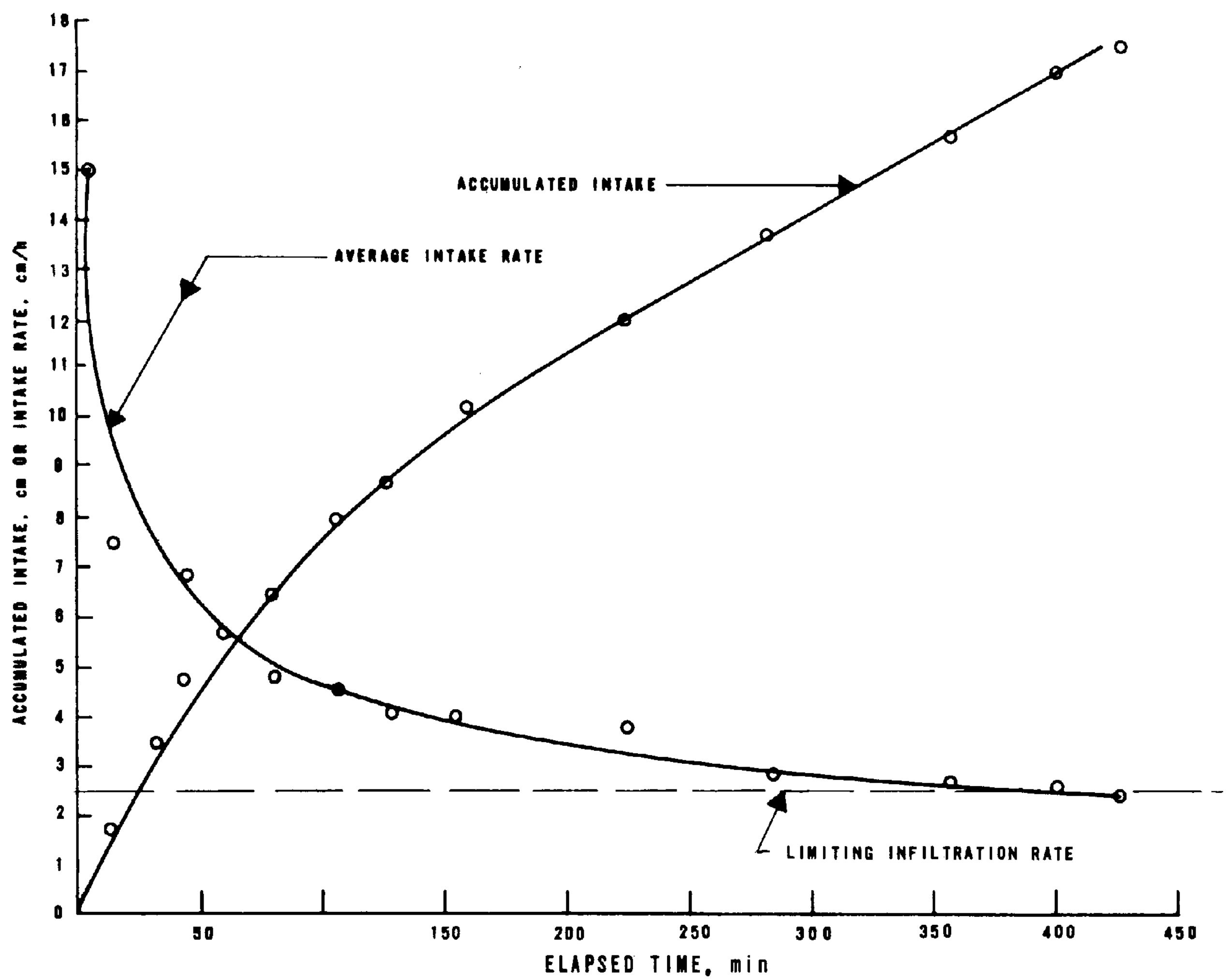

FIGURE B-3
INTAKE CURVES - INFILTRATION BASIN 1

2.5 cm/h (1 in./h). This was the minimum infiltration rate from the two tests and was used as the basis for design.

B.5 Determination of Wastewater Loading Rate

B.5.1 Preapplication Treatment Level

The existing treatment facilities are old and necessary repair work would not be cost effective. Therefore, new preapplication treatment facilities are needed. To consolidate the treatment facilities, Community B decided to locate the preapplication treatment facilities adjacent to the RI facilities at Site 1. Because Site 1 is close to the community, biological treatment prior to land treatment was appropriate (Section 5.3.1). The area experiences mild winter weather, making ponds the most cost-effective form of preapplication treatment.

The land available for preapplication treatment was somewhat limited; to minimize the pond area, an average depth of 3.6 m (12 ft) was selected. The pond design included surface aerators to be used periodically for odor control and to keep the pond from becoming entirely anaerobic. The pond was divided into three aeration cells for flexibility and reliability. A design detention time of 3 days was selected and adjustable weirs were included in each cell to allow wastewater withdrawal after 1 to 2 days if treatment efficiency is high or if the BOD:N ratio must be increased to promote denitrification during RI. The expected effluent quality from the aerated lagoons is 75 mg/L BOD5 and 90 mg/L SS. Because of the short detention time, the nitrogen content will remain at 50 mg/L and the ammonia nitrogen content will be approximately 20 mg/L.

B.5.2 Hydraulic Loading Rate

The annual hydraulic loading rate was designed to be within 10 to 15% of the limiting basin infiltration rate (Table 5-11 and Section 5.4). A median value of 12.5% was selected and the wastewater loading rate was calculated as follows:

$$
\begin{aligned}
& 12.5\% \times 2.5 \text{ cm/h} \times 0.01 \text{ m/cm} \\
& \quad \times 365 \text{ d/yr} \\
& \quad = 27.4 \text{ m/yr } (90 \text{ ft/yr})
\end{aligned}
$$

B.5.3 Hydraulic Loading Cycle

Because the renovated water will flow laterally or be drained into North Creek, nitrification or ammonium nitrogen removal is necessary during the months of May through October. To maximize nitrification, a loading cycle of 2 days of flooding alternated with 12 days of drying was selected (Section 5.4.2). Using this loading cycle and the assumed loading rate, the volume of water applied during each loading cycle is:

$$\frac{(2d + 12d)/\text{cycle}}{365\ d/yr} \times 27.4\ m/yr \times \frac{100\ cm}{m}$$

$$= 105\ cm/cycle\ (41.4\ in./cycle)$$

B.5.4 Effect of Precipitation on Wastewater Loading Rate

As shown in Table B-3, precipitation in Community B averages 111 cm/yr (3.6 ft/yr) and varies throughout the year from 5.5 to 15.9 cm/mo (2.2 to 6.2 in./mo). As mentioned in Section B.2.5, the wettest year in 10 would yield 137 cm (54 in.) of precipitation. This amount roughly corresponds to a maximum monthly precipitation of 20 cm/mo (8.0 in./mo). Adding maximum monthly precipitation to the average wastewater loading rate of 2.3 in/mo (7.5 ft/mo) resulted in a maximum monthly hydraulic loading rate of 2.5 m/mo (8.2 ft/mo). This combined loading rate is 13% of the test basin infiltration rate and, therefore, was acceptable (Section 5.4.1).

For land requirement calculations, the previously calculated wastewater loading rate (27.4 m/yr or 90 ft/yr) was used because precipitation is relatively insignificant most of the time.

B.5.5 Underdrainage

As discussed in Section 5.7.2, at RI sites where both the ground water table and the impermeable layer underneath the aquifer are relatively close to the soil surface, it may be possible to avoid lengthy mounding equations by using the following procedure:

1. Assume underdrains are needed.

2. Use Equation 5-4 to calculate drain spacing.

3. If the calculated drain spacing is reasonable (between 10 m and 50 m or 33 ft and 160 ft), drains should be used.

4. If the calculated spacing is less than 10 m, no mounding calculations are needed but the cost of the underdrains may cause the system not to be cost effective and may necessitate reconsideration of other sites identified during Phase 1.

5. If the calculated spacing is greater than 50 m, an evaluation of ground water mounding is necessary.

Because Site 1 is underlain by a relatively shallow impermeable layer, underdrains would be the appropriate drainage method. A drain depth of 3 m (10 ft) and an allowable ground water mound height above the drains of 0.6 m (2 ft) were assumed. Using Equation 5-4, drain spacing was calculated:

$$S = \left[\frac{4KH}{L_w + P}(2d + H)\right]^{1/2}$$

where S = drain spacing, m

K = horizontal hydraulic conductivity, m/d
= 3.4 m/d (Section B.4.2)

H = allowable height of the ground water mound above the drains, m
= 0.6 m

d = distance from drains to underlying impermeable layer, m
= 3 m

L_w = annual wastewater loading rate, m/d

$$= \frac{27.4 \text{ m/yr}}{365 \text{ d/yr}} = 0.075 \text{ m/d}$$

P = average precipitation rate, m/d

$$= \frac{1.11 \text{ m/yr}}{365 \text{ d/yr}} = 0.003 \text{ m/d}$$

$$S = \left(\frac{4 \times 3.4 \text{ m/d} \times 0.6 \text{ m}}{0.075 \text{ m/d} + 0.003 \text{ m/d}}[(2 \times 3 \text{ m}) + 0.6 \text{ m}]\right)^{1/2}$$

$$= 26 \text{ m (85 ft)}$$

Because this spacing is reasonable and will keep the mound from becoming a problem, additional mounding calculations were not necessary. Because the percolate collected in the underdrains will be discharged into North Creek, it was necessary to design the remainder of the system to meet the discharge requirements summarized in Table B-2.

B.5.6 Nitrification

To determine whether the proposed system could meet the summer ammonia nitrogen discharge requirements, the nitrification potential of the system was evaluated. First, the nitrogen loading rate was calculated as follows:

$$L_n = \frac{10 C_n L_w}{365}$$

where L_n = nitrogen loading rate, kg/ha·d

C_n = applied total nitrogen concentration, mg/L

L_w = annual loading rate, m/yr

$$L_n = \frac{10 \times 50 \text{ mg/L} \times 27.4 \text{ m/yr}}{365}$$

$$= 37.5 \text{ kg/ha·d } (33.5 \text{ lb/acre·d})$$

This loading rate is well within the range of nitrification rates reported under favorable temperature and moisture conditions (Section 5.2.2). Because nitrification is required only during summer months when temperatures are fairly high, temperatures at the RI system will be favorable for the required nitrification. Furthermore, the relatively short application periods and longer drying periods of the selected loading cycle will ensure favorable moisture conditions and should allow virtually complete nitrification within a relatively short soil travel distance (Section 5.4.2).

B.6 Land Requirements

B.6.1 Preapplication Treatment Facilities

The average liquid depth of the aerated pond was designed to be 3.6 m (12 ft), based on an average detention period of 3 days. An additional 1 m (3.3 ft) of freeboard was provided to allow the liquid depth to vary during peak flows and emergency conditions. Each pond cell berm was designed to have a 1:3 slope (vertical:horizontal) on both interior and exterior sides and to be 1.2 m (4 ft) wide on top. Thus, the total area required for the pond is approximately 1.7 ha (4.2 acres).

B.6.2 Infiltration Basins

The area needed for infiltration was calculated as follows:

$$A = (365\ Q)/(10^4\ L_w)$$

where A = area required, ha

Q = average wastewater flow, m^3/d

L_w = annual loading rate, m

$A = (365 \times 6{,}060\ m^3/d)/(10^4 \times 27.4\ m/yr)$
$= 8.1$ ha (19.9 acres)

B.6.3 Other Land Requirements

Additional land was required for berms around the infiltration basins and for access roads. Preliminary system layouts indicated that a total of about 14 ha (35 acres) would be required. This number was used for preliminary cost estimates; actual land requirements were developed during final system design.

B.7 System Design

B.7.1 General Requirements

A schematic of Community B*s RI system is shown in Figure B-4. The existing screening and grit removal facilities will be retained and used because they are necessary to protect the new pumping station.

A pumping station will be constructed at the site of the abandoned treatment facilities to pump the screened wastewater through a 30 cm (12 in.) force main to the treatment ponds. Three 3.14 m^3/min (830 gal/mm) pumps will be included. Two pumps operated together will be able to handle a peak flow of 9,090 m^3/d (2.4 Mgal/d). The third pump will be a standby. Standby power at the pumping station will be provided by a diesel generator. Distribution to the infiltration basins will be by gravity flow from the ponds.

Infiltration basins were located on the area having the most suitable soils. Because this area is relatively flat, very little grading was required and nearly equal-sized basins could be located adjacent to one another. The selected 14 day loading cycle required that at least 7 basins be constructed to enable dosing of at least one basin every 2 days. For this reason, the area having suitable soils was divided as shown in Figure B-5, with 7 basins ranging in size from 0.98 to 1.3 ha (2.4 to 3.2 acres).

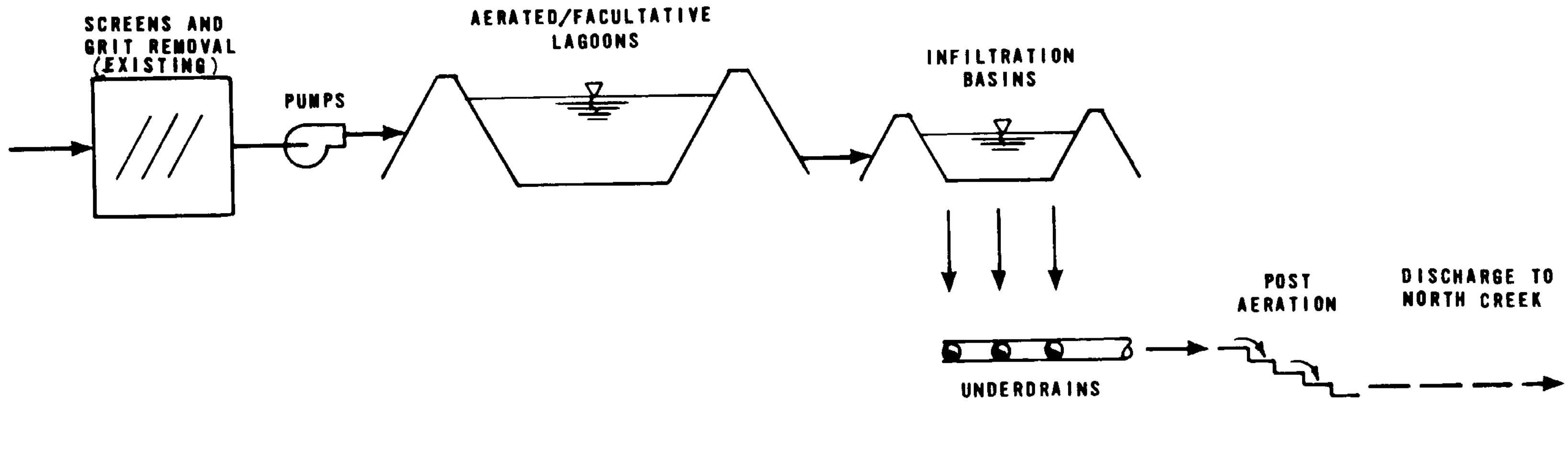

FIGURE B-4
COMMUNITY B RAPID INFILTRATION SYSTEM FLOWSHEET

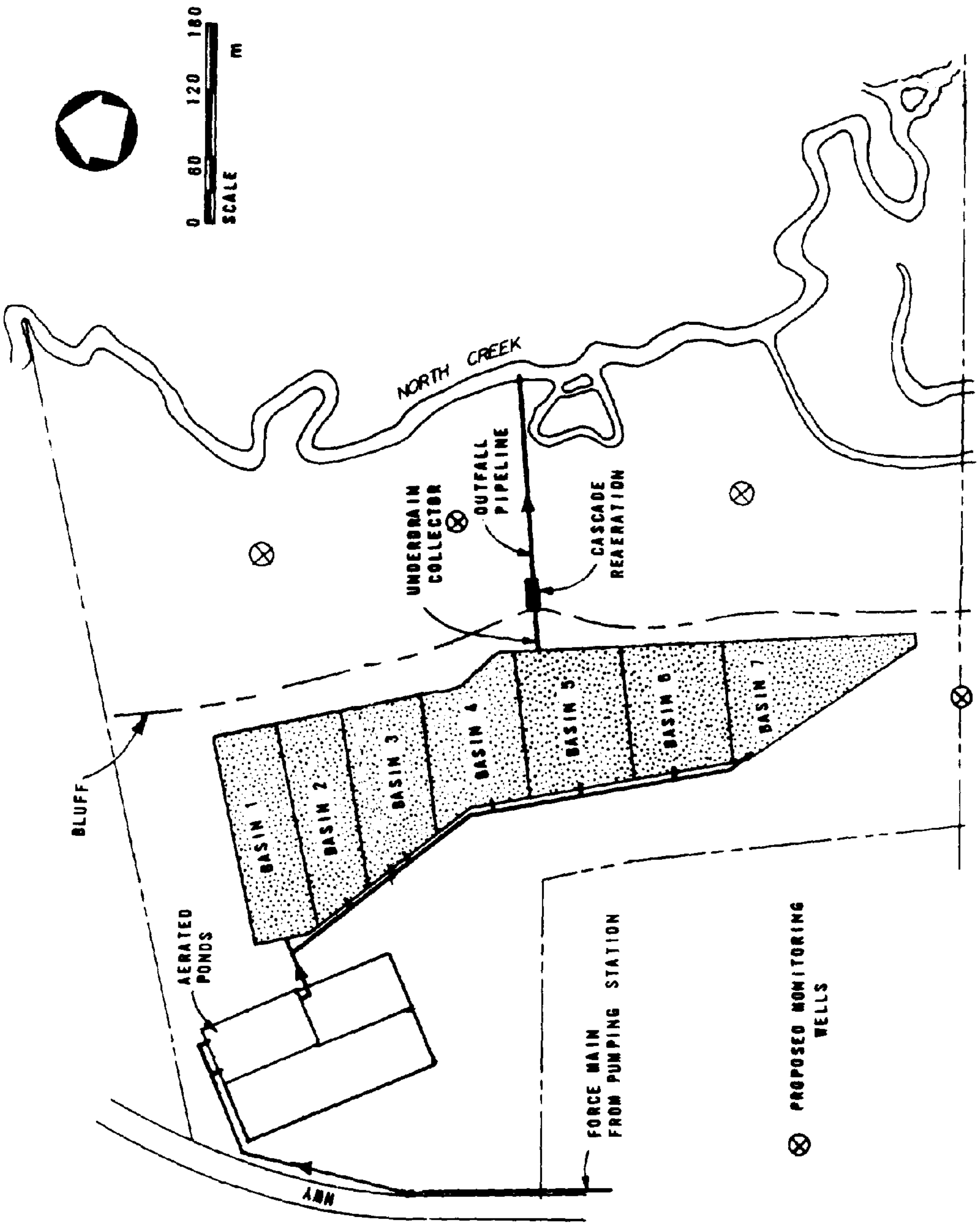

FIGURE B-5
COMMUNITY B SITE LAYOUT

To control the basin loading rate, adjustable overflow weirs were designed for each pond cell. During normal operation, the overflow weirs are to be set at the 3.65 m (12 ft) level of the pond (the average water depth). This means that the instantaneous wastewater flow to a basin at any time during a 2 day loading period will equal the wastewater flow just pumped into the pond. In other words, although the design average wastewater flowrate is 6,060 m^3/d (1.6 Mgal/d), up to 9,090 m^3/d (2.4 Mgal/d) may be delivered to each basin during peak flows (Section B.2.2). The peak wastewater application rate was calculated as follows:

$$R_{max} = \frac{Q_{max} \times 100 \text{ cm/m}}{A_{min} \times 10{,}000 \text{ m}^2\text{/ha} \times 24 \text{ h/d}}$$

where R_{max} = peak application rate, cm/h

Q_{max} = peak wastewater flow, m^3/d

A_{min} = basin area of smallest basin, ha

$$R_{max} = \frac{9{,}090 \text{ m}^3\text{/d} \times 100 \text{ cm/m}}{0.98 \text{ ha} \times 10{,}000 \text{ m}^2\text{/ha} \times 24 \text{ h/d}} = 3.86 \text{ cm/h}$$

In contrast, the average wastewater loading rate is:

$$R = \frac{Q \times 100 \text{ cm/m} \times N}{A_T \times 10{,}000 \text{ m}^2\text{/ha} \times 24 \text{ h/d}}$$

where R = average application rate, cm/h

Q = average wastewater flow, m^3/d

N = number of infiltration basins

A_T = total area covered by basins, ha

$$R = \frac{6{,}060 \text{ m}^3\text{/d} \times 100 \text{ cm/m } 7}{8.1 \text{ ha} \times 10{,}000 \text{ m}^2\text{/ha} \times 24 \text{ h/d}}$$

$$= 2.18 \text{ cm/h}$$

Comparing the peak and average application rates to the lowest measured basin infiltration rate of 2.54 cm/h or 1.0 in./h (Section B.4.3], it can be seen that during application, infiltration would exceed application at least half the time. Also, all of the water applied during a 1 day period would infiltrate during the same period.

Therefore, the basin depth necessary to allow up to 12 hours of flooding at the peak application rate:

$$D = (A_{max} - I) \times 12\ h$$

where D = maximum depth for wastewater, cm

A_{max} = basin area of largest basin, ha

I = limiting infiltration rate, cm/h

$$D = (3.86\ cm/h - 2.54\ cm/h) \times 12\ h$$
$$= 16\ cm\ (6.2\ in.)$$

The required total depth was found by rounding off D to 15 cm (6.0 in.) and by adding 30 cm (12 in.) of freeboard (Section 5.6.1). The resulting design basin depth was 45 cm (18 in.). This depth should provide more than adequate freeboard during normal operations and will provide a margin of safety for unexpected conditions and emergencies.

A typical slope, of 1:2 was selected for the sides of the berms, on both interior and exterior sides, and the width of each berm was set at 122 cm (48 in.). A single road around the outer edge of the basins was included with ramps into each basin for access. With these additions, the area covered by the infiltration basins was approximately 8.3 ha (20.5 acres), including 8.1 ha (19.9 acres) available for infiltration.

B.7.2 Underdrainage

Drain laterals and a collector drain were located as shown in Figure B-6. Drain lateral sizing will vary between 15 and 20 cm (6 and 8 in.), as recommended in Section 5.7.3. The collector drain will be 20 cm (8 in.) in diameter to ensure free flowing conditions. To meet the dissolved oxygen requirements for discharge to North Creek, the renovated water will be routed through a cascade aerator placed at the bluff west of North Creek.

B.8 Maintenance and Monitoring

B.8.1 Maintenance

Occasional cleaning and ripping of the basins will be required to maintain design infiltration rates (Section 5.8.2). Also, periodic maintenance of the ponds, pumping station, screens, and grit chamber will be necessary. A staff of two full-time employees should be able to handle all the operation and maintenance needs of Community B*s system (Section 2.3.3.1).

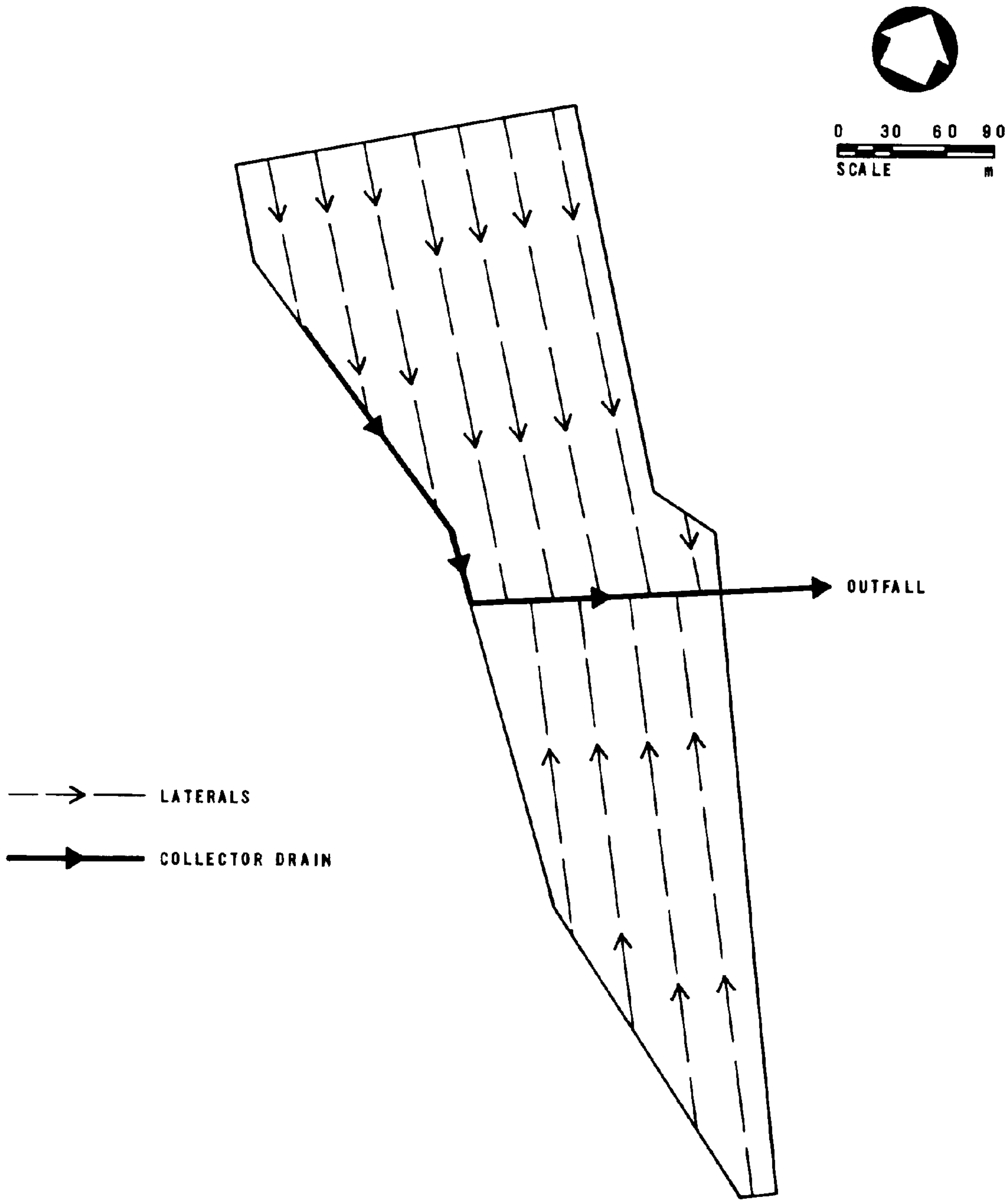

FIGURE B-6
UNDERDRAIN LOCATIONS

B.8.2 Monitoring

The renovated water will be monitored at the outfall for the parameters listed in Table B-2. Three monitoring wells to monitor ground water concentrations of ammonia nitrogen and total dissolved solids will be installed as shown in Figure B-5. An observation well will be installed between the bluff and Basin 4 to monitor ground water levels and evaluate underdrain performance.

B.9 System Costs

Total costs of Community B*s RI system are presented in Table B-7. Capital costs were estimated using the EPA report on Cost of Land Treatment Systems [l] . Costs were updated to October 1980 using the EPA Sewage Treatment Plant Construction Cost Index value of 397.2. Contractor*s overhead and profit are included in the cost estimates. The land was assumed to cost $4,900/ha ($2,000/acre). Operation and maintenance costs were estimated using the cost curves and current local prices for power and labor. Present worth was determined using an interest rate of 7-1/8% for 20 years.

B.10 Energy Budget

In Community B, energy required for land treatment will be used primarily to convey screened wastewater to the land treatment site. The amount of energy needed for this purpose can be estimated using the format presented in Section 8.6.2, as follows:

Elevation at treatment site	44 m (145 ft)
Elevation at pump station	32 m (105 ft)
Elevation difference 12 m (40 ft)	
Average flow	4,208 L/min
	(1,111 gal/min)
Assumed pumping system efficiency	40%
Pipeline diameter	30 cm (12 in.)
Pipeline length	2,680 m (8,000 ft)
Pipeline headloss	12 m (40 ft)
Total dynamic head	24 m (80 ft)

TABLE B-7
COST OF COMMUNITY B RI SYSTEM
Thousands of Dollars, October 1980

Capital costs	
Transmission pumping	290
Transmission main	289
Aerated lagoons	153
Field preparation	94
Infiltration basins	153
Underdrains	65
Cascade aerator	17
Outfall pipe	18
Monitoring wells	10
Service roads and fencing	52
Standby power	48
Laboratory equipment	24
Sewer rehabilitation	113
Land acquisition	273
Legal, administrative, engineering, interest, contingencies	332
Total capital costs	1,931
Operation and maintenance costs	
Annual labor	15
Annual materials	7
Annual power	17
Total operation and maintenance costs	39
Total project costs	
Total capital costs	1,931
Present worth of operation and maintenance	409
Total present worth of costs	2,340
Salvage value of land	(131)
Net present worth	2,209

Energy requirement (using Equation 8-2)	361,000 kWh/yr

The energy required for scarification is within the range of error of the estimated energy required to convey wastewater to the treatment site. For this reason, energy requirements for scarification are neglected. The energy required by the three cell pond would be approximately 395,000 kWh/yr. The total energy requirement of the system is 756,000 kWh/yr.

B.11 References

1. Reed, S.C., et al. Cost of Land Treatment Systems. U.S. Environmental Protection Agency. EPA-430/9-75-003. September 1979.